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# Review of Dynamic Testing of Highway Bridges

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# Technical Report Documentation Page

# Review of Dynamic Testing of Highway Bridges

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| Abstract: | A review is presented of the recent technical literature in the English language dealing with bridge dynamics in general and dynamic testing of highway bridges in particular. It is shown that several definitions have been used for the impact factor, because of which the same set of field test data may lead to widely varying estimates of the impact factor. In nearly all the reviewed references there is little or no justification for using a particular definition, suggesting that each of the various definitions was regarded as axiomatic, requiring no justification. It is also shown that there are other factors as well which may be responsible for misleading conclusions from the test data; these factors include the vehicle type, the vehicle weight, the transverse position of the vehicle with respect to the reference point, the differences in dynamic increment of strains and deflections, the presence of bearing restraint forces, and the roughness of the riding surface.<br><br>It was found that extensive dynamic testing of bridges has been undertaken in only a limited number of jurisdictions, namely Switzerland, U.S.A., Ontario, New Zealand, and the United Kingdom.<br><br>A preferred method of interpreting the field test data for obtaining a representative value of the impact factor is suggested. It is shown that the impact factor is not a tangible entity susceptible of deterministic validation. |
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# 1/ Introduction

Vehicles that are expected to cross a highway bridge during its lifetime are accounted for in the design of the bridge through a statistically applied design load and a certain prescribed fraction of it, which is traditionally referred to as the impact factor, and lately as the dynamic load allowance. The static design load is a tangible entity which can be derived from the static weights of actual and foreseen vehicles in such a way that the load effects induced by it in any bridge are representative, with a certain degree of reliability, of the load effects induced by these actual and foreseen vehicles. The impact factor, on the other hand, is an abstract entity which is supposed to account for the magnification of load effects in a bridge caused by the interaction of the vehicle and the bridge.

Despite its abstract nature, the impact factor has been used in the design of bridges for several decades. There have been numerous attempts to measure this elusive quantity in bridges through dynamic field testing. The purpose of this paper is to review, and draw some general conclusions from, the technical literature dealing with bridge dynamics in general, and dynamic testing of highway bridges in particular.

## 2/ Definition of Impact Factor

While the term "impact factor" is universally understood to mean the same quantity among bridge designers, there had been some objections to its use. For example, it is argued in the commentary to the Ontario Code (1979), that this term suggests that the dynamic loading of a bridge is merely due to the impactive action of wheels rather than being the result of several other factors. The Ontario Code refers to the impact factor as the "dynamic load allowance" (DLA).

As long ago as 1931, Fuller et al. suggested that the impact increment of dynamic force be defined as the amount of force, expressed as a fraction of the static force, by which the dynamic force exceeds the static force. Recognizing that the "impact increment of dynamic force" is not necessarily the same as the "impact increment of stress", the latter was defined as the amount of stress, expressed as a fraction of static stress, by which the actual stress due to moving loads exceeds the static stress.

Researchers interpreting test data from dynamic load tests have often used the term "dynamic increment" for the same quantity which has been defined by Fuller et al. (1931) as the impact increment of stress or which could have been defined as the impact increment of deflection. However, there is no uniformity in the manner by which this increment is calculated from test data. The different ways of calculating the dynamic increment can be explained conveniently with the help of Fig. 1, which has been constructed from data of an actual dynamic test with a two-axle vehicle on a right, simply supported plate girder bridge (Biggs and Suer, 1956). This figure shows the variation of both the dynamic and static deflections at mid-span of a girder with respect to time. The dynamic deflections were obtained when the test vehicle travelled on the bridge at normal speed, and the static deflections were obtained when the vehicle travelled at crawling speed so as not to induce dynamic magnification of deflections. Fig. 1 also shows the median deflections which were obtained by averaging consecutive peaks of dynamic deflections. As can be seen in this figure, the median deflections are not the same as the static deflections; however, a numerical procedure for filtering out the dynamic portion of the response can give the median responses that are fairly close to the static responses.

It may be noted that a fictitious scale of deflections has been introduced in Fig. 1 in order to facilitate the explanation regarding the interpolation of the test data. Referring to this figure, a notation is now introduced, as follows:

$\delta_{stat}$ = the maximum deflection under the vehicle travelling at crawling speed.

$\delta_{dyn}$ = the maximum deflection under the vehicle travelling at normal speed. This deflection is also denoted as $\delta_{max}$.

$\delta'_{stat}$ = the maximum deflection obtained from the curve of median deflections. Note, $\delta_{stat}$ and $\delta'_{stat}$ do not necessarily take place at the same load location.

$\delta_{min}$ = the minimum dynamic deflection in the vibration cycle containing $\delta_{max}$.

$\delta_1$ = the static deflection corresponding to $\delta_{max}$. As may be seen in Fig. 1, $\delta_1$ is not necessarily the maximum static deflection.

$\delta_2$ = the median deflection corresponding to $\delta_{max}$.

$\delta^*_s$ = the static deflection at the same location where $\Delta_1$ is recorded.

$\Delta_1$ = the maximum difference between dynamic and static deflections. As shown in Fig. 1, $\Delta_1$ does not necessarily take place at the same load position which causes either $\delta_{stat}$ or $\delta_{dyn}$.

$\Delta_2$ = the maximum difference between dynamic and median deflections.

$\Delta_3$ = the difference between dynamic and static deflections at the same load location which causes $\delta_{stat}$.

$\Delta_4$ = the difference between dynamic and median deflection at the same load location which causes $\delta'_{stat}$.

The above definitions can be more general if the word deflection is replaced by response.

From the data plotted in Fig. 1, the various deflection parameters are found to have the following values according to the fictitious scale from which units are deliberately omitted to maintain generality.

$\delta_{stat}$ = 10.0; $\delta_{dyn} = \delta_{max}$ = 12.3; $\delta'_{stat}$ = 9.9;

$\delta_{min}$ = 6.2; $\delta_1$ = 9.1; $\delta_2$ = 9.8; $\delta^*_s$ = 6.4;

$\Delta_1$ = 3.2; $\Delta_2$ = 3.1; $\Delta_3$ = 2.0; $\Delta_4$ = 2.0.

The various definitions are now described, many of which have been used in the past to obtain from the test data the dynamic increments, or similar parameters given different names. For the sake of convenience, all these different parameters will henceforth be referred to generically as dynamic amplification factors, and will be denoted by the symbol I.

**Definition 1**

According to the definition of impact increment of dynamic response of Fuller et al. (1931), the largest of I would be given by:

$$I = \frac{\Delta_1}{\delta^*_s} \tag{1}$$

Using this expression, the deflection data of Fig. 1 would lead to I = 0.500. It should, be noted, however, that this method is the result of a hypothetical and impractical extrapolation of a definition which, perhaps, was not intended for this purpose. This method has not been used to interpret test data in any of the references which were studied for this state-of-the-art report.

**Definition 2**

A commonly used variation of Definition 1 is that I is taken as the ratio of the measured instantaneous dynamic response to the maximum static response. Thus:

$$I = \frac{\Delta_3}{\delta_{stat}} \tag{2}$$

This definition of I has been used in the interpretation of data from several dynamic tests (e.g., Biggs and Suer, 1956; Edgerton and Beecroft, 1956; and AASHO, 1962) and in nearly all analytical studies (e.g., Scheffey, 1956; Fleming and Romualdi, 1961; Fenves et al., 1962; Wiriyahai et al., 1982).

According to Definition 2, the value of the impact factor obtained from the data given in Fig. 1 is 0.200.

**Definition 3**

When the static deflections are assumed to be the same as median deflections, Definition 2 of I changes to:

$$I = \frac{\Delta_4}{\delta'_{stat}} \tag{3}$$

This definition gives I the value of 0.202. It is noted that this definition does not seem to have been used by any of the references cited in this paper.

**Definition 4**

This definition was used in Switzerland to interpret test data from the dynamic bridge tests conducted during 1949 to 1965 (Cantieni, 1983a). According to this definition, the dynamic increment I is given by:

$$I = \frac{\delta_{max} - \delta_{min}}{\delta_{max} + \delta_{min}} \qquad (4)$$

It is noted that this definition of the dynamic increment was abandoned in Switzerland after 1965 in favour of Definition 5 which is given below. However, this definition has been used in New Zealand until fairly recently. For example, see Shepherd and Aves (1973) and Thurston (1983).

**Definition 5**

According to this definition, which has been used in Switzerland for tests conducted before 1945 and after 1965 (Cantieni, 1983a), the dynamic increment I is given by:

$$I = \frac{\delta_{dyn} - \delta_2}{\delta_2} \qquad (5)$$

using which, the data of Fig. 1 gives I = 0.255.

**Definition 6**

A variation of Definition 5 would be when the static response corresponding to the maximum dynamic response is taken as the same as the median response obtained from the dynamic test data. In this case, I is given by:

$$I = \frac{\delta_{dyn} - \delta_1}{\delta_1} \qquad (6)$$

leading to a value for I of 0.352.

This definition has been extensively used to interpret the results of many dynamic tests on bridges in Ontario (e.g., see Csagoly et al., 1972; Dorton, 1976; King et al., 1978; Campbell et al., 1979; and Bakht, 1980 and 1981).

**Definition 7**

For interpreting the data from some dynamic tests conducted in Ontario (Wright and Green, 1962; and Billing, 1982b), the following expression was used for obtaining I:

$$I = \frac{\delta_{dyn} - \delta'_{stat}}{\delta'_{stat}} \qquad (7)$$

For the specific case under consideration, the value of I is then found to be 0.242.

**Definition 8**

If the actual static responses are used instead of median responses, the following variation of Eq. (7) is obtained:

$$I = \frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \qquad (8)$$

This definition gives I a value of 0.230 for the data of Fig. 1.

## 3/ Comparison of Various Definitions

Table 1 contains the values of I obtained by the various definitions from the same set of data. It can be seen in this table that the values of I are all different and range from 0.2 to 0.5.

The various references studied for this state-of-the-art report are listed chronologically in Table 2, together with the definition used therein to compute I. In nearly all of the references there is little or no discussion or justification for using a particular definition of the dynamic amplification factor. This seems to suggest that each of the various definitions was regarded as being axiomatic and requiring no justification. Yet the variety of results given in Table 1 confirm that the definition of I is far from being axiomatic. What can be regarded as axiomatic, however, is the definition of the amplification factor for the response at a given instant. According to this definition, $I = \Delta / \delta_s$, where $\Delta$ is the difference between the static and dynamic responses at the instant under consideration, and $\delta_s$ is the corresponding static response.

The axiomatic definition of amplification factor given above is used, justifiably, in all the analytical studies; it is, however, of little use in bridge design because its value changes with time and load position. What is required for design purposes is a single value of the amplification factor using which maximum dynamic response can be computed from the maximum static response, so that:

$$\delta_{dyn} = \delta_{stat} (1 + I) \qquad (9)$$

The values of $\delta_{dyn}$ obtained by using $\delta_{stat}$ of 10.00 and the values of I given by the various definitions are also given in Table 1. Ideally, the use of the amplification factor according to Eq. (9) should have returned the same value of $\delta_{dyn}$ which was measured in the field, i.e., 12.30. It can be seen in Table 1 that none of the definitions has given the correct value of $\delta_{dyn}$. Definition 8, the equation for which (Eq. (8)) is, in fact, the same as Eq. (9) does, however, give the correct value of $\delta_{dyn}$. It is interesting to note that the apparently logical Definition 8 has not been used in any of the references given in Table 2.

At a cursory glance, it may seem strange that such a variety of definitions emerge from the process of selecting a single governing value from the values of I which are obtained by an axiomatic definition. A careful scrutiny of the problem will readily reveal that the diversity in the definitions of I from measured responses, is the consequence of the fact that (a) the static response of a bridge is not necessarily the same as the median response obtained from the dynamic test data; and (b) the maximum static and dynamic responses do not always take place under the same load position (e.g., see Biggs and Suer, 1956). If the static and median responses were identical and the

maximum static and dynamic responses took place simultaneously, the diversity of definitions of I would disappear and Definitions 2 through 8 would all give the same value of I for a given set of data.

It is emphasized that Definition 1 is of academic interest only, and has not been used in real life to calculate the dynamic amplification factors.

## 4/ Factors Responsible for Misleading Conclusions

The technical literature reports a fairly large scatter in the values of the dynamic amplification factor of a given response, even when the bridge and the vehicle are the same. From these observations it can readily be concluded that the dynamic amplification factor is not a deterministic quantity. To obtain a single value of this factor for design purposes, it is necessary, as is shown later in the paper, to know the statistical properties of the scatter of data, in particular the mean and variance of the amplification factor. The various parameters which can influence the statistical properties of the amplification factors computed from the test data are discussed in the following. If not accounted for carefully, these parameters can influence misleadingly the way in which the measured data are interpreted.

### Vehicle Type

It already known that the dynamic amplification factor for a bridge is influenced significantly by the dynamic characteristics of the vehicle with respect to those of the bridge. Despite this fact, most dynamic tests on bridges have been conducted with specific test vehicles. The data from such tests cannot, for obvious reasons, be regarded as representative of actual conditions. The amplification factors obtained from tests with only specific test vehicles can only provide a qualitative insight into the problem of bridge dynamics. They should not be used to obtain the final single value of the impact factor which is to be used in calculations for design or evaluation. A representative value of the impact factor can be calculated realistically only when data are gathered under normal traffic and over relatively long periods of time.

### Vehicle Weight

Several researchers have concluded from observed data that the dynamic amplification factor due to a vehicle decreases with the increase of vehicle weight (e.g., see Fuller et al., 1931; Shepherd and Aves, 1973; Billing, 1982b; and Thurston, 1983). It can be appreciated in light of this information, that the amplification factors corresponding to lightly loaded vehicles, which are irrelevant to the design load effects, are likely to weigh the data unduly on the higher side. The data corresponding to lightly loaded vehicles should not be used at all in the calculation of the impact factor, unless, of course, the impact factor is sought specifically for lighter vehicles, as it may be for the evaluation of the load carrying capacity of existing substandard bridges.

**Vehicle Position With Respect To Reference Point**

The cross-section of a three-lane slab-on-girder bridge is shown in Fig. 2. The bridge has five girders, all of which are instrumented for dynamic response measurement; it carries a vehicle in the far right-hand lane so that girders 4 and 5 carry the vehicle load directly. In this case, girders 1 and 2, being remote from the applied load, will carry a very small portion of the static load. Yet the dynamic amplification of the small portion of the static load carried by these two girders is likely to be fairly large. It has been observed by several researchers that the dynamic amplification factor at a reference point well away from the load can be larger than that for a reference point directly under the load. Clearly, the former amplification factor has no relevance as far as the maximum static load effects are concerned at the cross-section of a bridge.

Cantieni (1983a) uses a parameter $\alpha$ to define the relative position of the vehicle with respect to that of the reference point; this parameter is defined by:

$$\alpha = \frac{D}{H + 1.25} \tag{10}$$

Where D is the distance (in metres) in the transverse direction between the reference point and the nearest line of wheels, and H is the depth of construction of the bridge (also in metres) at the instrumented cross-section, the number 1.25 in the denominator is the half-width of the vehicle in metres. When $\alpha$ is less than 1.0, the reference point is assumed to lie within the direct influence zone, in which case the dynamic test data is considered to be relevant. In a case when $\alpha$ is greater than 1.0, the data are ignored, being of no consequence.

Figure 3 identifies the zone of direct influence defined by Cantieni (1983a) It can be seen that this zone is obtained simply by drawing 45° lines on the bridge cross-section from the outer line of wheels of the test vehicle. This limit on the zone of influence cannot be regarded as being applicable to all bridges as it does not take into account the transverse load distribution characteristics of the bridge. It may be more appropriate to take into consideration, only the data from that reference point at which the static load response is the maximum across the cross-section. Admittedly, this approach necessitates a relatively large number of reference points, one of which should always be at or near the position of maximum load effect.

The statistical properties of the dynamic amplification factor, computed from the test data, can be regarded as realistic only if the extraneous data from outside the zone of influence are excluded

from consideration. It is surprising that the attempt to exclude such extraneous data has been explicitly mentioned in only a few of the references cited in this paper.

## Deflection Versus Strain Measurements

The response of a bridge in dynamic tests is conventionally measured by means of vertical deflections. The measurement of strains is resorted to only when the measurement of deflections is found to be difficult (e.g., see Billing, 1982a and Cantieni, 1981). The dynamic amplification factors obtained by both approaches are assumed to be applicable with equal validity to all responses including deflections, bending moments and shear forces. Strictly speaking, the practice of regarding the amplification factors as being applicable for other responses is not correct. It has been demonstrated convincingly by the AASHO test (1962) that, under similar conditions, the dynamic amplification factors computed from measured deflections are always greater than the corresponding factors computed from measured strains. The same conclusion has also been reached by others (e.g., see Wright and Green, 1962).

In the absence of any bearing restraint, girder strains are directly proportional to girder moments. Consequently, for bridges without appreciable bearing restraint, the dynamic amplification factors computed from strains can be regarded realistically as being the same as those for girder moments. However, it is still not established whether the amplification factors obtained from either deflections or strains could be applied validly for other responses such as longitudinal shears and transverse moments.

The problem is further complicated by the presence of bearing restraint effects which, as shown below, are significant in most of the slab-on-girder bridges.

## Bearing Restraint Effects

With the exception of AASHO tests (1962), the measured frequencies of tested bridges were nearly always found to be much greater than the calculated frequencies. This observation leads to the conclusion that the flexural stiffness of bridges is nearly always greater than the calculated stiffness. The reason for this apparent discrepancy has been attributed, wrongly, in many of the references to (a) a very high modulus of elasticity of concrete; (b) interaction with the main components of secondary components such as horizontal bracings and barrier walls; and (c) the presence of a high degree of composite action between the girders and deck slab, even when they do not have any mechanical shear connection between them.

King et al. (1978) performed a pull-down and quick-release type of test on a slab-on-girder bridge, both before and after the casting of the concrete barrier walls, which has been suggested by many to stiffen bridges considerably. The fact that the measured natural frequency of the bridge remained almost unaffected by the addition of the barrier walls, clearly suggests that in this bridge the barrier walls do not have a major influence on the overall flexural frequency of the bridge.

Unintentional composite action in an apparently non-composite bridge can also not always be regarded as a major factor to stiffen the bridge. This is so because, even those slab-on-girder bridges have been found stiffer than shown by calculations which have mechanical shear connectors between the deck slabs and girders and in which full composite action has been considered in the calculations.

Biggs and Suer (1956) calculated the effective flexural rigidity of the tested bridges by using both the measured midspan static load deflections and the measured frequencies.

When it was found that the two procedures gave different flexural rigidities, it was concluded that, "although this cannot be completely explained, it is possible that the causes are a greater participation of secondary elements and a greater degree of frictional restraint at the ends of the span". This is, perhaps, the first time in recent literature that attention has been paid to the presence of bearing restraint effects as being the possible cause of bridges being stiffer than shown by calculations.

Recent tests by Bakht (1988a, b and c) have confirmed that fairly large bearing restraint forces develop in slab-on-girder bridges at the interfaces of girders and the surfaces they rest upon. These large bearing restraint forces have been found to stiffen the bridges appreciably.

The observed frequencies of composite steel and prestressed concrete bridges tested in the AASHO study (1962) were in good agreement with the calculated values which were obtained by using the measured cross-sections and other properties of the actual bridges. It is noted that, in these bridges, the girders were supported by rocker-type bearings, which prevented the development of substantial bearing restraint forces. Since, in the absence of substantial bearing restraint forces, it was possible to get a good match between the observed and calculated frequencies, it can be concluded with confidence that, in those bridges in which the observed frequencies are larger than the calculated ones, the bearing restraint forces are substantial.

The overall horizontal movement of the bottom flanges of continuous girders are much smaller than those of corresponding single-span bridges. This fact has been confirmed by the observations of Eyre and Tilly (1977), who found that continuous-span bridges have smaller damping values than single-span bridges. It can be readily appreciated that the smaller movement at the bearings of continuous-span bridges leads to smaller dissipation of energy at the bearings and, consequently, to smaller damping values. The bearing restraint forces in continuous-span bridges, however, can still be large enough to affect their frequencies of vibration which have been found to be smaller than calculated (e.g., see Campbell et al., 1979 and Lee et al., 1985).

The presence of bearing restraint forces induces compressive strains in the bottom flanges of the girders; consequently the girder strains do not remain directly proportional to the girder moments. In such cases it is very doubtful whether dynamic amplification factors obtained from girder strains can apply exactly to girder moments.

The inclusion of bearing restraint forces in the interpretation of measured dynamic strains is still not a practical proposition. This problem, however, still needs a careful study.

# 5/ Miscellaneous Observations

## Quality of the Riding Surface

It is mentioned in nearly every reference dealing with bridge dynamics that the roughness of the riding surface of a bridge and its approaches has a very significant influence on the dynamic magnification of load effects in the bridge. It is usual in dynamic bridge tests to account for the riding surface irregularities by placing a wooden plank of appropriate thickness in the path of the test vehicle. As might be expected, the dynamic amplification factors corresponding to such tests are always larger than those which correspond to similar tests without the wooden plank.

The practice of performing dynamic tests by creating a temporary irregularity in the riding surface of even well maintained bridges has been defended on the basis that even a well maintained bridge can have a sudden irregularity introduced in the riding surface, for example, as a result of an object being dropped accidentally from a vehicle, or the accumulation of packed snow, etc. It should be noted, however, that in bridges with more than one lane the design loading for failure almost invariably corresponds to the rare event of exceptionally heavy vehicles being simultaneously present in two or more lanes of the bridge. The probability of such a rare event happening, at the same time as the formation of an accidental irregularity in the riding surface of an otherwise well-maintained bridge, is indeed so small as to be negligible.

The commentary to the second edition of the Ontario Code (1983) suggests that a high value of the dynamic load allowance should be used if the approach is unlikely to be paved for extended periods of time, or if the expansion joints between the superstructure and approach pavements are not expected to be flush with the roadway. Taking a cue from this suggestion, it may be appropriate to test such bridges by placing a plank in the path of the test vehicle. For well-maintained bridges, on the other hand, there does not seem to be any justification for adopting this approach.

## Multi-lane Loading

It has been observed in many references that the dynamic amplification factor for more than one vehicle is always less than that for a single vehicle, and the out-of-phase dynamic actions of the various vehicles are usually cited as the reason for this phenomenon. It seems appropriate that the value of the impact factor should decrease with the increase in the number of loaded lanes. The Ontario Code in its first edition (1979), specified a separate set of multi-presence reduction factors for the dynamic load allowance (DLA). These factors were much smaller than the corresponding

factors for static loading. The separate reduction factors for DLA were abandoned in favour of the same factors for both the static loading and DLA in the second edition of the Code (1983). Jaeger and Bakht (1987) have shown that the multi-presence factor for combined static and dynamic loading, $m_f$ is approximately given by:

$$m_f = \frac{m_{fs} + m_{fd}\,DLA}{1 + DLA} \qquad (11)$$

where $m_{fs}$ and $m_{fd}$ are the multi-presence reduction factors for static and dynamic loadings, respectively.

**Continuous Bridges**

A reference point on a continuous-span bridge may undergo responses corresponding to both positive and negative moments due to the passage of a vehicle. Fig. 4 shows the deflections at midspan of an outer span of a three-span continuous bridge. The dynamic deflections shown in this figure are due to a vehicle as it moves from left to right. As can be seen in this figure, the deflections have three distinct regimes: (a) when the vehicle is on the instrumented (i.e., left-hand) span; (b) when the vehicle is on the middle span; and (c) when the vehicle is on the right-hand span.

When the vehicle is on the instrumented span, the definition of the dynamic amplification factor remains the same as for a simply-supported bridge. However, when the vehicle is on the middle span, causing negative moments at the instrumented section, the definition of the amplification factor becomes even more complex than it is for simply-supported spans. Billing (1988) suggests the following definition of the dynamic amplification factor for such a case:

$$I = \frac{\bar{\delta}_{max} - \bar{\delta}_S}{\delta_S} \qquad (12)$$

where the notation is as shown in Fig. 4. Billing (1988) notes that the above definition of I gives a realistic lower bound of total dead and live load stresses as they may be required for fatigue calculations.

# 6/ Testing Activity, World Wide

In the main, extensive dynamic testing of highway bridges for determining the dynamic amplification factors has been found to be undertaken in only a few jurisdictions in the world. A brief review of such testing activity around the world is instructive. It is noted that Hudson (1977) provides a state-of-the-art review of the dynamic testing of structures, but this review is limited to structures other than bridges.

## Switzerland

In Switzerland, all bridges having spans greater than 20 m must be dynamically tested. Cantieni (1981, 1983a and b and 1984) presents a wealth of data from the 60 years of dynamic testing of highway bridges in Switzerland. As shown in Table 2, he used Definition 5 for obtaining the dynamic amplification factors from the test data.

The provisions of the Ontario Highway Bridge Design Code (1983) are attested to by the conclusions drawn from the Swiss data as presented by Cantieni (1983a). Nevertheless, it should be noted that the very extensive data presented by Cantieni still cannot be regarded as being representative of actual conditions, since the tests in Switzerland were conducted only with two-axle test vehicles.

## U.S.A.

Other than the Swiss tests, the first series of dynamic tests on highway bridges, reported in the English technical literature in recent times, appears to be those by Biggs and Suer (1956); Foster and Oehler (1956); Hayes and Sbarounis (1956); and Edgerton and Beecroft (1956). Out of these references, that due to Biggs and Suer appears to be especially significant since the authors have attempted to explain methodically the mechanics of interaction between the dynamic systems of the bridge and vehicle.

The AASHO test series (1962), in which 18 experimental slab-on-girder bridges with single lanes were tested under both static and dynamic loads, appears to be the only systematic and extensive study of bridge dynamics conducted in the United States. However, as Fenves et al. (1962) point out, the tests on these experimental bridges can be regarded only as tests intermediate between small-scale laboratory tests and full-scale field tests.

After the AASHO tests, the only series of field tests conducted in the U.S.A. to determine dynamic amplification factors in highway bridges seem to be those which were undertaken primarily as weigh-in-motion studies (e.g., see Moses, 1979; and Moses et al., 1984). It seems that the data on amplification factors obtained from these tests have not yet been published separately.

There have been other dynamic tests on bridges conducted in the United States (e.g., Varney, 1981; Salane et al., 1981; Doyle and Reid, 1982; and Douglas et al., 1982), but these did not deal with the measurement of impact factors.

**Ontario**

Ontario seems to be the only place in the world where several systematic field studies have been undertaken in recent times with a view to understanding the interaction of bridge and vehicle dynamic systems in general, and to obtain the dynamic amplification factors in particular. The first of these studies, in which 52 bridges were considered, is reported by Wright and Green (1962). This report is perhaps the first, and certainly one of the most significant works on bridge dynamics in recent times. Many of the conclusions presented in this report have withstood the test of time and remain valid today.

The second series of tests in Ontario was undertaken on eleven different continuous-span bridges (Csagoly et al., 1972). These tests were conducted using specific test vehicles, and, therefore, all the data obtained from these tests cannot be regarded as being representative of actual conditions. However, what also emerged from these tests was the far-reaching conclusion that the dynamic amplification factors are relatively large when the fundamental frequency of the bridge lies in the 2 to 5 Hz range, which is also the usual range for the fundamental frequencies of commercial vehicles. This conclusion is at the basis of the dynamic load allowance provisions of the Ontario Code.

The third series of tests in Ontario, which also constitutes one of the most significant studies of highway bridge dynamics ever undertaken, was conducted by Billing on 27 different bridges. Some information regarding these tests is given by Billing (1982a and b). The rest of the information is still unpublished.

Besides the systematic studies mentioned above, several individual dynamic tests have also been conducted in Ontario as, for example, those by Cicci and Csagoly (1974); Dorton (1976); King et al. (1978); Rainer and Pernice (1979); and Bakht (1980 and 1981).

A review of the various dynamic tests conducted in Ontario has been provided by Green (1977); Campbell et al., (1979); Billing and Green (1984); and Agarwal et al. (1988).

**New Zealand**

Seventeen different highway bridges were tested for their dynamic response in New Zealand. These tests are described by Shepherd and Aves (1973; Shepherd and Sidwell (1973); and Wood and Shepherd (1979). Like most other dynamic tests, the bridges in New Zealand were also tested with specific test vehicles, which limits the applicability of the conclusions drawn therefrom. It is also difficult to utilize directly the dynamic amplification factors from the New Zealand tests because the definition of the amplification factors used for interpreting the data from these tests, namely Eq. (5), is no longer used by most researchers in the world.

Individual dynamic tests, other than those reported above, have also been conducted in New Zealand. Two such tests are reported by Thurston (1983).

**United Kingdom**

The studies undertaken in the United Kingdom were mainly concerned with either the measurement of the dynamic amplification of wheel loads (e.g., Leonard et al., 1974; and Page, 1976), or with dynamic characteristics of bridges other than the dynamic amplification factors (e.g., Leonard, 1974; Eyre and Tilly, 1977; and Wills, 1977).

# 7/ Analytical Studies

Several methods of analysis have been developed in the past to analytically solve the problem of the dynamic interaction of a bridge and a particular vehicle (e.g., Scheffey, 1956; Fleming and Romualdi, 1961; Fenves et al., 1962; Gupta, 1980; Wiriyachai et al., 1982; and Inbanathan and Wieland, 1987). Notwithstanding the ability of these methods to model accurately the problem of bridge and vehicle interaction, their use in the design office is likely to remain very limited. This is so because, unlike static loads, there is no design vehicle which can be used to obtain the upper bound of the dynamic magnification factors induced by all vehicles expected to traverse a bridge in its lifetime. It seems that the problem can be solved analytically only by analyzing a bridge under the action of a huge number of vehicles of different dynamic characteristics. This, clearly, is not a practical proposition.

# 8/ Recommended Procedure

From a survey of the technical literature dealing with dynamic testing of highway bridges, a preferred procedure emerges naturally for obtaining, through a test, a representative value of the impact factor for single vehicles which can be used realistically in the load capacity evaluation of an existing bridge. The various steps involved in this preferred and recommended procedure are given in the following.

## Instrumentation

For obtaining the dynamic amplification factors for longitudinal moments, it is preferable to measure strains rather than deflections. In slab-on-girder bridges, strains can be measured conveniently at the bottom flanges of the girders, even if the girders are of concrete, in which case special strain gauges or strain transducers (e.g., see Wagner, 1981) may have to be used. Care should be exercised in selecting a crack-free zone for instrumentation of concrete components.

The instrumented section should preferably be parallel to the support lines for both skew and right bridges, especially when the construction incorporates girders. There should be enough reference points on the instrumented section to enable the maximum static load response to be picked up across the section for any realistic vehicle position. For picking up the maximum static response in a skew slab type of bridge, the reference points may have to be arranged in a different pattern than a straight line.

## Calibration Test

It is desirable to perform both static and dynamic tests on the bridge with vehicles of known weights and configurations. The "static" test can be performed either under a stationary vehicle positioned at preselected locations, or under a vehicle moving at crawling speed of less than about 10 km/h. If the latter procedure is adopted, it may still be necessary to filter out the dynamic responses in order to obtain the static load responses. However, in this case the filtered responses can be expected to be very close indeed to the actual static load responses.

The static load responses should be used to establish approximately the maximum strain or deflection levels at the various reference points which correspond to the different levels of the evaluation loading. The correlation between the maximum static response and the median response should be established from the calibration tests. It is recalled that the median responses are obtained from the dynamic responses by filtering out the local peak values.

**Elimination of Extraneous Data**

At a given instrumented cross-section, the data corresponding to only a single reference point should be considered when computing the dynamic magnification factor due to a vehicle pass on the bridge. This reference point should be the one at which the maximum static, or median, value of the response is recorded for the vehicle pass under consideration.

As discussed earlier, the dynamic amplification factors corresponding to light vehicles, being relatively on the high side, tend to bias the data. It is, therefore, necessary that the data considered for developing the statistics of the dynamic amplification factor corresponds to the weight class of the design or evaluation vehicle. This can be achieved as follows.

The maximum response at a reference point due to the design or evaluation vehicle, can be readily obtained by extrapolation of the data obtained from the calibration tests. Let this maximum response be denoted as $\varepsilon$. As shown in Fig. 5, the observed maximum static load response can be divided into a number of strata, with each stratum representing the vehicles of a certain class of weight which relates to the load effects in the bridge rather than the gross vehicle weight. It is recommended that the observed data from the dynamic test should be divided into various groups, depending upon the division of the maximum static load effects, of the kind shown in Fig. 5. For example, the dynamic amplification factors corresponding to the design or evaluation vehicle should be obtained only from that data for which $\delta'_{stat}$ lies within $0.9\varepsilon$ and $1.1\varepsilon$.

**Method for Obtaining Dynamic Amplification Factor**

From a purely logical standpoint, the appropriate definition for computing the dynamic amplification factor from measured responses would appear to be Definition 5, described earlier. However, this definition requires that a bridge be tested under the same vehicle separately for both dynamic and static effects. Such a requirement is obviously not realistic when data are being collected under normal traffic. In such a case, it is suggested that the next best definition is Definition 7. This definition can give fairly reliable results, especially if it is found from the calibration test that the maximum static load response $\delta_{stat}$ is close in magnitude to the corresponding maximum median response $\delta'_{stat}$. The case in which the two maximum responses are significantly different from each other is rather rare. For such bridges, Definition 7 can still be used, but only after making appropriate adjustments for the difference between $\delta_{stat}$ and $\delta'_{stat}$.

It is noted that when a vehicle is longer than the length of the influence line of the instrumented component, the static response at a reference point due to the moving vehicle may not be smooth,

i.e., it may have "static oscillations". For such cases the obtaining of the median responses by automatic filtering is made especially difficult when the period of "static oscillations" matches with the period of dynamic oscillations.

**Calculation of Impact Factor**

The term "impact factor" is used here to denote that single value of the dynamic amplification which is used in the calculations for the design or evaluation of the bridge. It is noted that, as mentioned earlier, the impact factor is also referred to as the "dynamic load allowance".

Simply because of the scatter in their values, the dynamic amplification factors computed from the dynamic test data cannot be used directly as the impact factor. Neither should an upper-bound value of the amplification factor be used as the impact factor, since this is likely to prove overly conservative. A logical approach to computing a representative value of the impact factor from the test data would be to cater for the variability of the amplification factor in the same way as it is done for the variability of the static loads. One procedure proposed in the commentary to the second edition of the Ontario Code (1979) and reported by Billing (1982), can be used to achieve this goal. According to this procedure, the specified value of the impact factor, $I_S$, depends not only upon the statistics of the amplification factor but also upon the live load factor specified in the Code and the safety margin to which the Code is calibrated. The expression for obtaining $I_S$ is as follows:

$$I_S = \frac{\bar{I}\,(1 + vs\beta)}{\alpha} \qquad (13)$$

where: $\bar{I}$ = the mean value of the dynamic amplification factor.

$v$ = the coefficient of variation of the dynamic amplification factor, being the ratio of standard deviation and mean.

$s$ = separation factor for dynamic loading, which has been found to have a value of 0.57.

$\beta$ = the safety index which, typically, has a value of about 3.5 for highway bridges.

$\alpha$ = the live load factor.

It is noted that when the distribution of mean largest vehicle weights is log normal, it may be more appropriate to use the following expression for obtaining the specified value of this impact factor:

$$I_s = \frac{\bar{I}}{\alpha} e^{v s \beta} \qquad (14)$$

The use of the live load factor in Eqs. (13) and (14) requires some discussion. As pointed out by Agarwal (1988), the use of the live load factor in determining the specified value of the impact factor is open to question, as the live load factor alone does not define the variability of the static loads. This variability is accounted for by both the live load factor and the specified static loads. For example, it is possible to get the same live load effects by simultaneously doubling the live load factor and halving the specified static loads. In this case, the specified value of the impact factor would be reduced incorrectly if Eqs. (13) and (14) were used.

Ideally, $\alpha$, in Eqs. (13) and (14), should account for the variability of static loads, rather than being equal to the live load factor. It is recommended that, in the absence of more rigorous analysis, the value of ˇ should be taken as 1.4, which is also the live load factor as specified in the Ontario Code. Using the numerical values of the various variables, Eqs. (13) and (14) can be written as:

$$I_s = 0.71\ \bar{I}\ (1 + 2.0v) \qquad (15)$$

$$I_s = 0.71\ \bar{I}\ e^{2.0v} \qquad (16)$$

It should be noted that for $\bar{I}$ and v to be representative of actual conditions, there should be a sufficient number of values of the amplification factor obtained from the test data.

# 9/ Conclusions

From a survey of the technical literature dealing with the dynamic testing of highway bridges, it has been found that there is a general lack of consistency in the manner in which the test data are interpreted to obtain the values of the dynamic amplification factor. A preferred method of calculating these factors has been proposed.

It has been shown that the impact factor is not a tangible entity susceptible of deterministic evaluation; it can be accounted for in the design and evaluation of bridges only by a probabilistic approach. Despite being obtained from field data by a preferred procedure, the impact factor still remains an abstract entity which should, in general, be treated only as a design convenience.

A recommended procedure for obtaining a design value of the impact factor by testing a highway bridge under dynamic loads has been developed.

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**Table 1** Values of dynamic amplification factor I and computed maximum dynamic deflection

| **Definition No.** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** |
|---|---|---|---|---|---|---|---|---|
| Value of I | 0.500 | 0.200 | 0.202 | 0.330 | 0.255 | 0.352 | 0.242 | 0.230 |
| Value of $\delta_{dyn}$ from Eq. (9) | 15.00 | 12.00 | 12.02 | 13.33 | 12.55 | 13.52 | 12.42 | 12.30 |

*Note: Maximum value of measured dynamic deflection is 12.30.*

**Table 2** List of references on dynamic behaviour of bridges and the definitions for determining I

| No. | Reference | Subject | Definition used for Computing I |
|---|---|---|---|
| 1. | Fuller et al., 1931 | Philosophical essay | 1 |
| 2. | Biggs & Suer, 1956 | Field testing | 2 |
| 3. | Foster & Oehler, 1956 | Field testing | * |
| 4. | Hayes & Sbarounis, 1956 | Field testing | * |
| 5. | Edgerton & Beecroft, 1956 | Field Testing | 2 |
| 6. | Scheffey, 1956 | Analytical | 2 |
| 7. | Fleming & Romualdi, 1961 | Analytical | 2 |
| 8. | AASHO, 1962 | Field testing | 2 |
| 9. | Fenves et al., 1962 | Field testing & analytical | 2 |
| 10. | Wright & Green, 1962 | Field testing | 7 |
| 11. | Varney, 1971 | Field testing | * |
| 12. | Csagoly et al., 1972 | Field testing | 6 |
| 13. | Shepherd & Aves, 1973 | Field testing | 4 |
| 14. | Cicci & Csagoly, 1974 | Field testing | * |
| 15. | Leonard, 1974 | Testing procedure | * |
| 16. | Leonard & Granger, 1974 | Vehicle testing | * |
| 17. | Dorton, 1976 | Field testing | 6 |
| 18. | Page, 1976 | Vehicle testing | * |
| 19. | Green, 1977 | State-of-the-art report | 7 |
| 20. | Wills, 1977 | Analytical | * |
| 21. | Eyre & Tilly, 1977 | Field testing | * |
| 22. | Hudson, 1977 | State-of-the-art survey | * |
| 23. | King et al., 1978 | Field testing | 6 |
| 24. | Campbell et al., 1979 | Field testing | 6 |
| 25. | Ontario Code, 1979 | Commentary | * |
| 26. | Rainer & Pernice, 1979 | Field testing | * |
| 27. | Wood & Shepherd, 1979 | State-of-the-art essay | 4 |
| 28. | Gupta, 1980 | Analytical | 5 |
| 29. | Cantieni, 1981 | Field testing | 5 |
| 30. | Salane et al., 1981 | Field testing | * |
| 31. | Wheeler, 1981 | Philosophical essay | * |
| 32. | Bakht, 1981 | Field testing | 6 |
| 33. | Douglas & Reid, 1982 | Field testing | * |
| 34. | Billing, 1982 | Testing procedure | * |
| 35. | Billing, 1982 | Field testing | 7 |
| 36. | Wiriyachai et al., 1982 | Analytical | 2 |
| 37. | Ontario Code, 1983 | Commentary | * |
| 38. | Thurston, 1983 | Field testing | 4 |
| 39. | Cantieni, 1983a | Field testing | 5 |
| 40. | Cantieni, 1983b | Field testing | 5 |
| 41. | Cantieni, 1984 | Field testing | 5 |
| 42. | Billing & Green, 1984 | State-of-the-art essay | * |
| 43. | Lee et al., 1985 | Field testing | * |
| 44. | Inbanthan & Wieland, 1987 | Analytical | 5 |
| 45. | Agarwal et al., 1988 | State-of-the-art essay | * |
| 46. | Steiner et al., 1988 | Field testing | * |

* *not applicable*

**Figure 1** Midspan deflection under a moving vehicle load

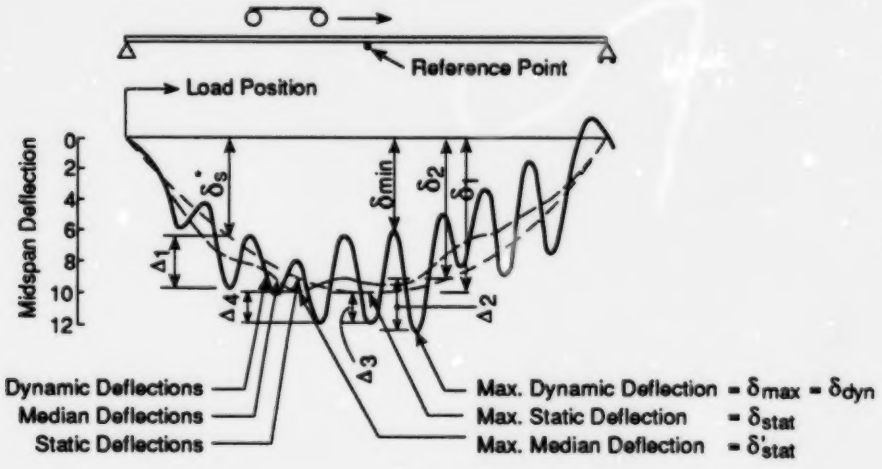


**Figure 2** Cross-section of a three-lane bridge

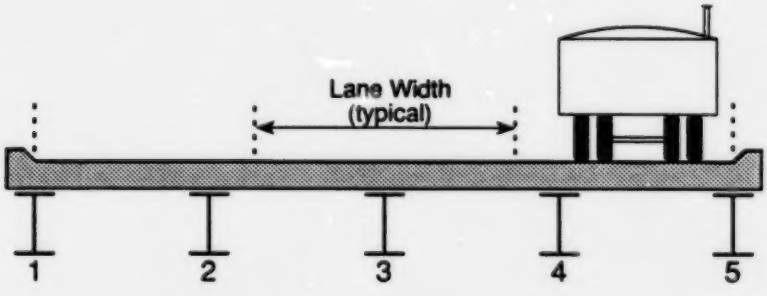

**Figure 3** Definition of the zone of direct influence (Cantieni, 1983)

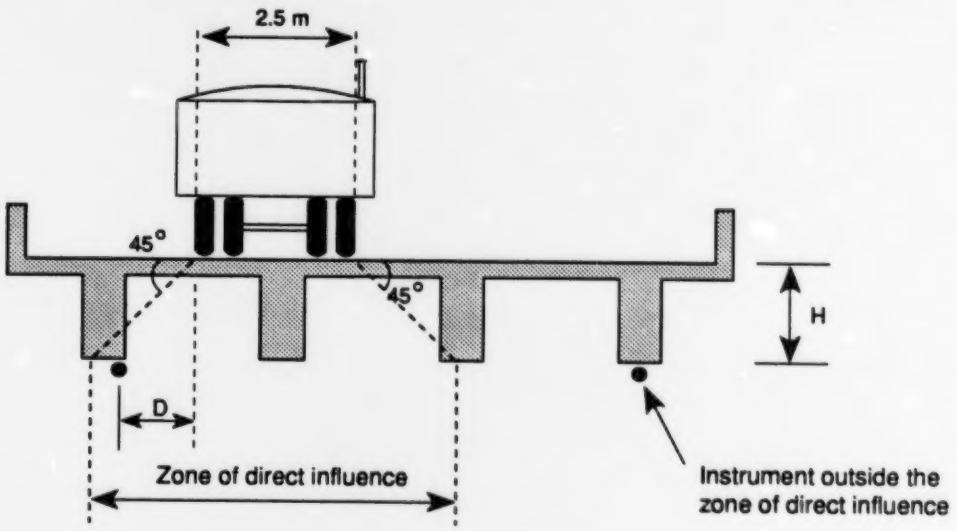


**Figure 4** Dynamic response of a three-span continuous girder

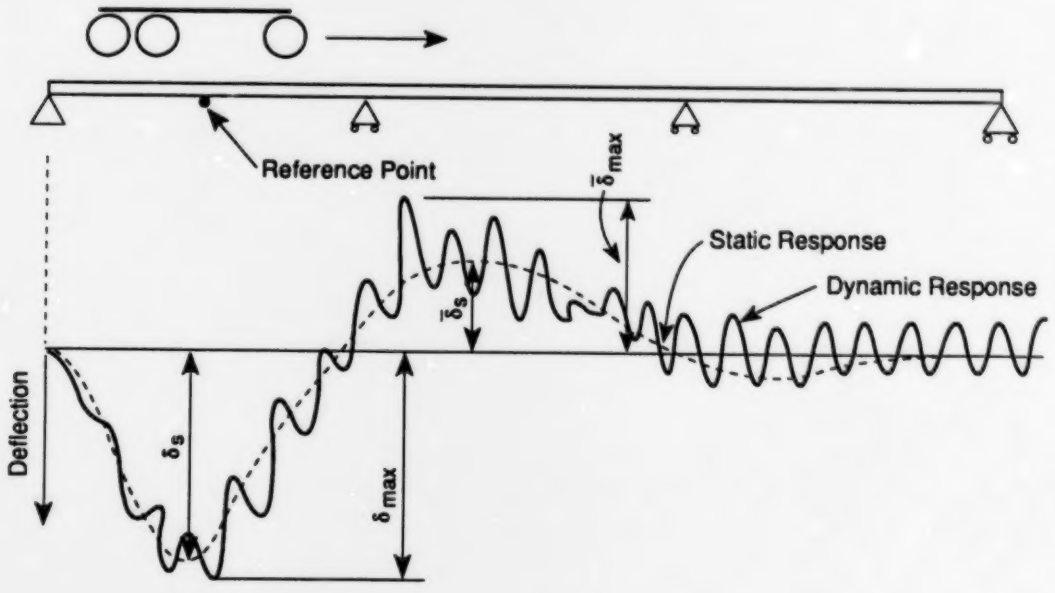

**Figure 5** A scheme for classifying vehicles